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ORIGINAL ARTICLE

Using simulated hydrologic response to revisit the 1973 Lerida Court landslide

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Abstract Hydrologically driven mass wasting in the form of landslides on steep slopes is a worldwide occurrence. High-profile events in, for example, Brazil, Chile, the Philippines, Puerto Rico, and Venezuela during the last three decades all clearly illustrate, based upon significant losses of life and property, that hydrologically driven slope instability in developed (urban) areas can be a major geologic/environmental hazard. The focus of this study is the 1973 hydrologically driven Lerida Court landslide in Portola Valley, CA, USA. Physics-based hydrologic-response simulation, with the comprehensive Integrated Hydrology Model, was employed to forensically estimate the spatiotemporal pore pressure distributions for the Lerida Court site. Slope stability, driven by the simulated pore pressure dynamics, was estimated for the Lerida Court site with the infinite slope/Factor of Safety approach. The pore pressure dynamics for the Lerida Court site were reasonably captured by the hydrologic-response simulation. The estimated time of slope failure for the Lerida Court site compares well with field observations. A recommendation is made that hydrologically driven slope stability estimates including variably saturated subsurface flow be standard protocol for development sites in steep urban settings.

Keywords Landslide · Integrated Hydrology Model · InHM · Hydrologic-response simulation · Slope stability · Factor of Safety

Abbreviations

FS	Factor of Safety
InHM	Integrated Hydrology Model
USA	United States of America

Introduction

Hydrologically driven slope instability in urban environments can be a problem (Alexander 1989). For example, landslides in the San Francisco Bay Area, USA during the major storms of January 1982 resulted in 24 fatalities and millions of dollars in property damage (Smith and Hart 1982). One of these failures occurred in a steep hollow behind Oddstad Boulevard in Pacifica, CA, USA (Shlemon et al. 1987). The Oddstad debris flow was particularly devastating in that three children lost their lives when their home was rapidly overrun.

It is important to recognize that even slow moving slope failures can endanger human lives and the built environment (e.g., Hilley et al. 2004; Petro et al. 2004; Vlcko 2004). The 1973 Lerida Court landslide is an example of a slow moving urban slope failure. The Lerida Court site is located approximately 7-km southwest of Stanford University in Portola Valley, CA, USA. Figure 1 shows the

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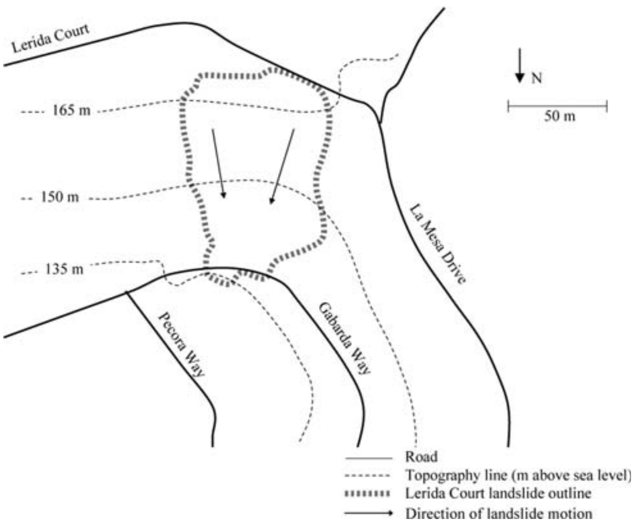


Fig. 1 Location of the Lerida Court landslide (Holzhausen 1974)

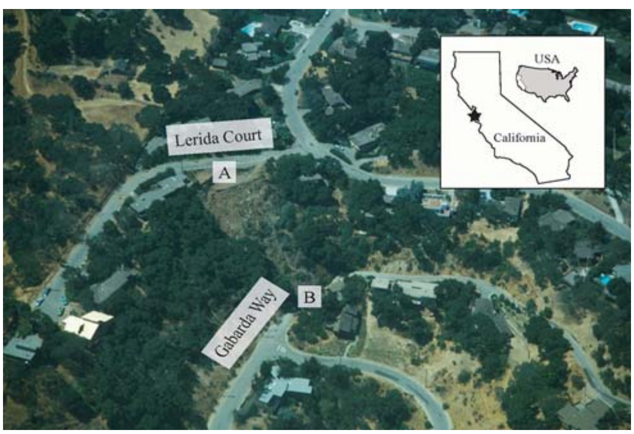


Fig. 2 Aerial photograph of the Lerida Court landslide (August 1973)

orientation of the Lerida Court landslide. The major cause of the Lerida Court landslide was the buildup of hydrologically driven subsurface pore pressures after record winter rainfalls (BeVille 2007). Figure 2 is an aerial photograph of the landslide showing a large scarp just off of Lerida Court (near the top of the failure, site A) and the debris blocking Gabarda Way (at the bottom of the failure, site B). It is important to note that a home on Lerida Court was removed from the site once the failure was underway. Two other homes were (are) located along Lerida Court within the vicinity of the landslide (see Fig. 2). One home was (is) located along Gabarda Way at the bottom of the landslide (see Fig. 2). Ultimately, only one home on Lerida Court was demolished; the three other homes in the area of the landslide sustained repairable damage (Holzhausen 1974). Figure 3 shows the doomed home on Lerida Court just prior to its removal. Figure 4 shows the blocked Gabarda Way. Two retaining walls were built on the landslide

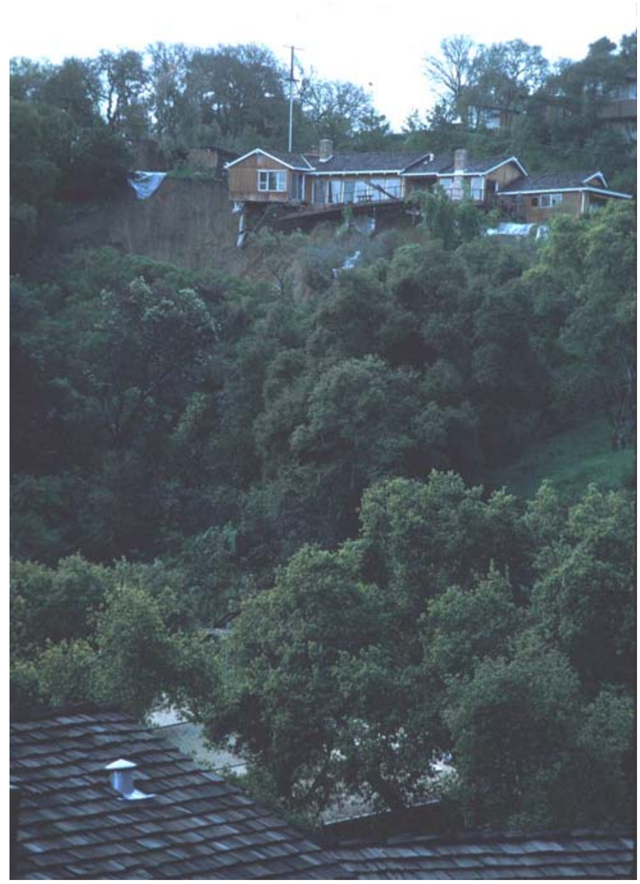


Fig. 3 Doomed home on Lerida Court prior to removal in March 1973 (see *A* in Fig. 2 for location)

Fig. 4 Debris blocked Gabarda Way in January 1974 (see *B* in Fig. 2 for location)

site to stabilize the failure. The lower wall (at Gabarda Way) was designed to hold back a significant mass of unconsolidated sediment (see Fig. 2) and prevent the buildup (via drains) of subsurface pore pressures.

Today, more than 35 years after the failure, the site remains stable. Recently, the possibility of constructing a new home on Lerida Court (at or near the site of the home removed in 1973) has been considered. Obviously, in lieu of the known risk, the spectacular view overlooking the San Francisco Bay is seductive. A good foundation for understanding the potential impact from a new development at the Lerida Court site is a process-based characterization of the 1973 failure.

The reason why the Lerida Court site failed can be addressed qualitatively with anecdotal information. For example, in 1969, cracks were observed in the Lerida Court road surface at the top of (what would be) the landslide site. The construction of a home on Lerida Court which was eventually torn down was finished in June 1970 and immediately incurred foundation cracking and structural distress, indicating incipient instability, which is consistent with the 594 mm of rain in the 1970 calendar year (FBLA 1973). Then in February 1973, after record winter rainfall, new cracks were observed in the same section of the then repaved Lerida Court. It is well established that persistent road surface cracking is a good surrogate for subsurface movement (e.g., Sas et al. 2008). Therefore, at first blush, the Lerida Court site appears to have been a loaded gun, which was set-off with the buildup of subsurface pore pressures. The objective of this study was to conduct a forensic style, simulation-driven assessment of the hydrologic response that led to the initiation of the 1973 Lerida Court landslide. This effort does not address the fully coupled processes of hydrologic response and slope deformation, which would require a level of site characterization and state variable observations that are unavailable for the Lerida Court site. Pore pressure results from the hydrologic-response simulations are used to drive a relatively simple slope stability model. The approach pursued here is to evaluate the small-scale hydrology of a specific landslide-prone site, which is different than the more frequently employed regional-scale landslide assessments used in urban areas (e.g., Keefer et al. 1987; Montgomery et al. 2001).

Methods

The physics-based Integrated Hydrology Model (InHM) used in this study was designed, developed, and tested by VanderKwaak (1999). The Darcy–Buckingham flux, $\vec{q}$ ($\mathrm{MT^{-1}}$), as employed in the variably saturated subsurface flow component of InHM, is given by

$$\vec{q} = -k_{\mathrm{rw}} \frac{\rho_{\mathrm{w}} g}{\mu_{\mathrm{w}}} \vec{k} \nabla(\psi + z) \tag{1}$$

where k_{rw} is the relative permeability (−), ρ_{w} is the density of water ($\mathrm{ML^{-3}}$), g is the gravitational acceleration ($\mathrm{LT^{-2}}$), μ_{w} is the dynamic viscosity of water ($\mathrm{ML^{-1}T^{-1}}$), $\vec{k}$ is the permeability vector ($\mathrm{L^2}$) ($k = K\mu_{\mathrm{w}}/\rho_{\mathrm{w}} g$), K is the saturated hydraulic conductivity ($\mathrm{LT^{-1}}$), z is the elevation head (L) and ψ is the pressure head (L). When combined with conservation of mass, Eq. 1 yields Richards' equation of 3D variably saturated flow in the subsurface, which (in InHM) is fully coupled with the diffusion-wave approximation of the shallow-water equations for flow on the land surface. The control-volume finite-element method is employed to implicitly solve the governing surface and subsurface flow equations as a single system using Newton iteration with efficient and robust sparse matrix methods (VanderKwaak 1999). The innovative linking of the variably saturated surface and subsurface continua allows InHM to simulate the four-principal runoff generation mechanisms, which are Horton overland flow (i.e., infiltration excess), Dunne overland flow (i.e., saturation excess), subsurface storm flow and groundwater, without an a priori specification of the dominant process. InHM has been successfully employed for several catchment-scale hydrologic-response simulations (e.g., VanderKwaak and Loague 2001; Ebel et al. 2007a, 2008, 2009; Heppner et al. 2007; Mirus et al. 2007; Heppner and Loague 2008; Mirus et al. 2009).

The slope stability model used in this study, based upon the infinite slope/factor of safety approach, is expressed as (see Selby 1993):

$$\mathrm{FS} = \frac{(c' + \Delta c) + [\gamma z \cos^2 \beta - p] \tan \phi}{\gamma z \sin \beta \cos \beta} \tag{2}$$

where FS is the factor of safety (i.e., the sum of resisting forces divided by the sum of driving forces) (−), c' is the intrinsic cohesion of the soil ($\mathrm{ML^{-1}T^{-2}}$), Δc is root cohesion ($\mathrm{ML^{-1}T^{-2}}$), γ is the unit weight of the soil ($\mathrm{ML^{-2}T^{-2}}$), z is the depth from the surface to the shear plane (L), β is the slope angle (°), p is the pore water pressure ($\mathrm{ML^{-1}T^{-2}}$) and ϕ is the angle of internal friction (°). It should be noted that the Δc term is the contribution to the shear strength term that resists failure and only applies to the base of the slide mass in Eq. 2. The relationship between the pressure head in Eq. 1 and the pore water pressure in Eq. 2 is given by:

$$p = \rho_{\mathrm{w}} g \psi. \tag{3}$$

With Eq. 2, a slope is estimated to be unstable (subject to failure) when the value of FS is ≤ 1.0. The application of the infinite slope model in this study is similar to that of Dutton et al. (2005) and Mirus et al. (2007) with separate FS estimates made across the failure plane, driven by individual pore pressures rather than a single average pore pressure. This pointwise application protocol assumes that the infinite slope assumptions are valid at every point on the failure surface.

The assumptions of the limit equilibrium infinite slope method presented in Eq. 2 include that peak strength is fully mobilized simultaneously along the failure surface (Duncan 1996), displacements occur as rigid bodies (Wong 1984) and that force inclinations can be specified as invariant for the entire period prior to failure (Duncan 1996). The infinite slope method only estimates the initiation of failure (i.e., when forces driving failure exceed forces resisting failure). Equation 2 also underestimates the resisting forces because it does not include 3D effects, such as lateral forces, including the lateral contribution of root strength along the margins of the slide mass (Stark and Eid 1998). Although not as rigorous as the hydrologic-response model used in this effort (i.e., InHM), the infinite slope method provides a first-order understanding of the hydrologic controls of landslide initiation at the Lerida Court site.

Site description

Construction of the Ladera neighborhood, where the Lerida Court site is located, began in the late 1950s. Home foundations were constructed by cut and fill methods, whereby the hillside was cut to provide a flat area for road construction and the displaced soil formed artificial terraces for building construction (Holzhausen 1974). Additional fill material was brought in by San Mateo County contractors (Holzhausen 1974), reaching an estimated 36,500 metric tons of added material at the landslide headscarp at Lerida Court. A site analysis of the engineering geology after the Lerida Court landslide indicated that failure to remove paleolandslide debris (Hoexter 1975) combined with not benching the fill into the slope to prevent sliding contributed to the instability of the slope. The vegetation at the landslide site before the slope failure included shrubs and small trees.

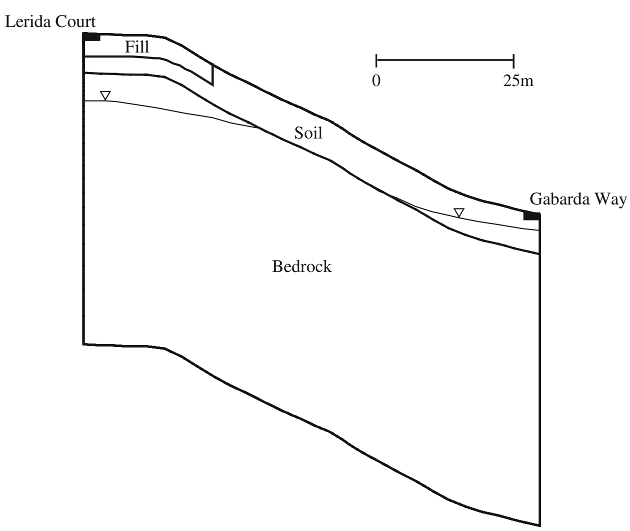


Fig. 5 Average vertical cross section (long profile) of the Lerida Court site before the failure (after FBLA 1973). The cross section is approximately representative of a cross section through *A–B* in Fig. 2

Figure 5 is a vertical cross section showing the hydrogeologic units of the landslide site prior to the failure. The soil layer (i.e., the soil from cut and fill methods) comprises the majority of the landslide mass, with the imported fill material also present in the slide mass. The characteristic curves (i.e., hydraulic conductivity and soil–water content as functions of pressure head) for the unsaturated near surface were represented by the van Genuchten (1980) method. The van Genuchten parameters and saturated hydraulic conductivity values for the fill and soil units were estimated based upon soil-textural class information in FBLA (1973), from the Carsel and Parrish (1988) catalog. Tables 1 and 2 provide, respectively, characteristics of the boundary-value problem and the geotechnical parameter values used in this study. The parameter values of each

Table 2 Average values used for the infinite slope/factor of safety analysis

Parameter	Value
Intrinsic cohesion (kN m^{-2})	1.5[a]
Root cohesion (kN m^{-2})	2.0[a]
Unit weight of soil (kN m^{-3})	19.6[a]
Depth (surface to failure plane) (m)	7.0[b]
Internal angle of friction (°)	35.0[c]
Slope angle (°)	21.6[b]

[a] Selby (1993), [b] FBLA (1973), [c] Holtz and Kovacs (1981)

Table 1 Characteristics of the Lerida Court boundary-value problem used for the InHM simulation

Characteristic	Fill	Soil	Bedrock	Road
Texture	Sandy clay loam[a]	Silt loam[a]	Sandstone[a]	Asphalt
Thickness (m)	3.5[a]	7.0[a]	50.0	0.3
Saturated hydraulic conductivity (ms^{-1})	5×10^{-6}[b]	1×10^{-6}[b]	1×10^{-8}[c]	1×10^{-13}

The values for gravitational acceleration, the density of water and the dynamic viscosity of water were taken as 9.8 ms^{-2}, 1,000 kg m^{-3} and 0.001138 kg $m^{-1}s^{-1}$, respectively

[a] FBLA (1973), [b] Carsel and Parrish (1988), [c] Freeze and Cherry (1979)

hydrogeologic unit for the site are assumed to be homogenous and isotropic. The failure plane is assumed to be at the soil/bedrock interface.

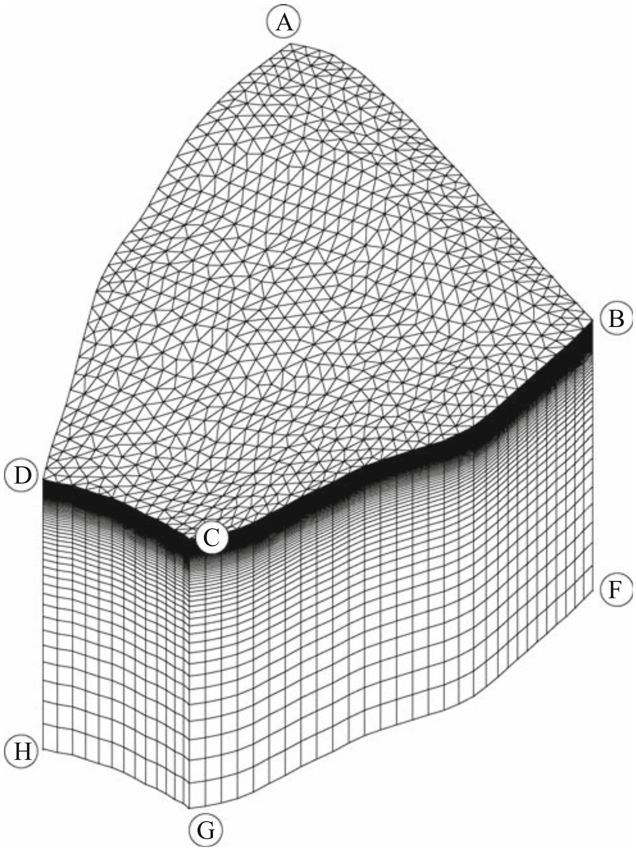
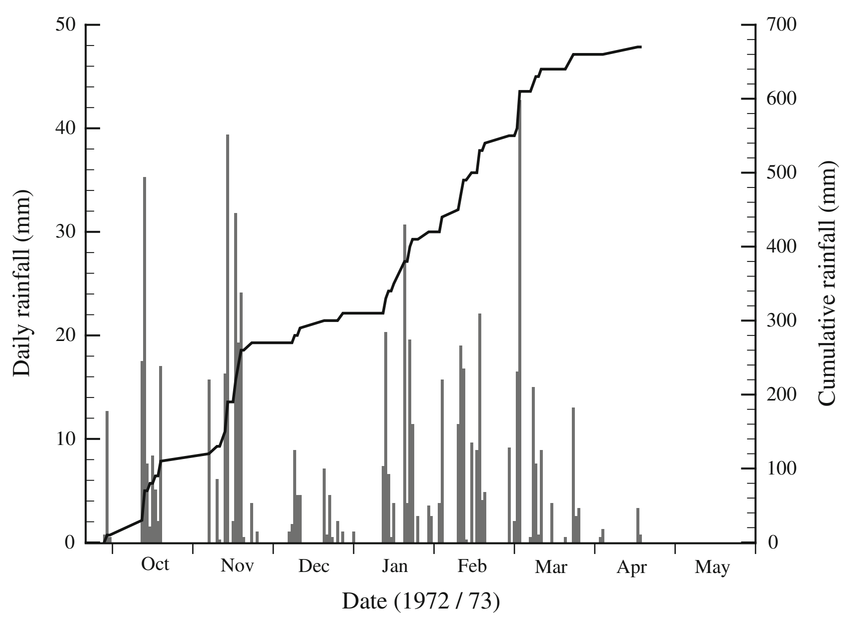


Fig. 6 Finite-element mesh used for the hydrologic-response simulations (note, point E is hidden)

Figure 6 shows the finite-element mesh used in this study for the hydrologic-response simulations. The mesh, generated from the pre-slide surface topography, consists of 67,336 nodes and 124,500 elements. The vertical nodal discretization of the mesh is finest (0.3 m) near the (unsaturated) surface and increases logarithmically with the depth into the (saturated) subsurface. With reference to Fig. 6, the boundary conditions associated with the mesh are no flow on the ADEH and BCFG sides (note, both sides follow topographic noses, taken here as subsurface flow divides), no flow on the up-gradient ABEF face, specified head outside the domain [i.e., local sink, see Heppner et al. (2007) and Ebel et al. (2008)] on the down-gradient CDHG face, no flow on the EFGH base, and a temporally variable specified flux (rainfall) on the ABCD surface. The start of the long-term simulation is 1 October 1972, which was chosen to coincide with the new water year (when rainfall typically begins at the field site). Initial conditions for the model are specified for this time (i.e., 1 October). The initial conditions were set via simulation (with InHM) consisting of 8 months of gravity-driven drainage from a nearly saturated system, which is physically consistent with the seasonal rainfall patterns of the Mediterranean climate at the Lerida Court site. The Lerida Court hillslope typically reaches peak saturation in February or early March; the observed rainfall record for 1972 shows zero rainfall during June through the end of September. The water table position at the start of the 8-month continuous simulation, supported by field observation (FBLA 1973), is shown in Fig. 5. The extended rainfall season hyetograph associated with the Lerida Court landslide is shown in Fig. 7. The rain gage is located at the Junior Museum in Palo Alto, approximately 9-km southwest of the Lerida Court site.

Fig. 7 Rainfall used for the hydrologic-response simulations [for data see http://www.ncdc.noaa.gov/oa/ncdc.html (verified 23 Feb 2009)]



The average rainfall for the nearly 57-year period of record (i.e., 1953–2009) at Palo Alto for the months of January and February was 79 and 84 mm, respectively (http://www.ncdc.noaa.gov/oa/ncdc.html). In 1973, the January rainfall totaled 132 mm (167% of the average value); the February rainfall totaled 167 mm (200% of the average value). On a yearly basis, 1973 was the fourth wettest calendar year in Palo Alto during the period of record, trailing only the major El Niño years of 1982, 1983 and 1998; heavy rain during these 3 years also caused landslides in the San Francisco Bay Area (Spiker and Gori 2003). Widespread slope failures occurred in the San Francisco Bay Area in the 1972–1973 winter rainy season in response to the heavy rainfall, causing $10 million in damage, at 1973 dollar values (Taylor et al. 1975).

Results

Figure 8 shows snapshots of the spatial distribution of simulated pore pressures for the Lerida Court site at the start of the simulation (Fig. 8a) and at 6 months elapsed time (Fig. 8b). The snapshots in Fig. 8 were made at the soil/bedrock interface. Inspection of Fig. 8, in comparison with the rainfall time series in Fig. 7, shows that the simulated pore pressures buildup significantly after the winter rainfall. Figure 8 also shows, as should be expected, that the simulated pore pressures are lower up-gradient (Lerida Court) and higher down-gradient (Gabarda Way). Most of the simulated subsurface flow is diverted down-gradient above the soil/bedrock permeability contrast with seepage onto Gabarda Way. The buildup of the simulated pore pressures at 6 months (Fig. 8b) is a function of less water leaving than entering the system during the simulation period. The available storage in the near surface is reduced as the water table rises. It should be pointed out that for the Lerida Court site, unlike (for example) for the Coos Bay slope failure site (see Ebel et al. 2007a, b), there are no observed hydrologic-response data with which to evaluate the performance of InHM (or any other model). Figure 8 shows the locations for six simulated observation points (at the soil/bedrock interface) that are used to report results from the slope stability analyses.

Figure 9 shows the factor of safety (FS) estimates for the Lerida Court site, driven by the InHM simulated pore pressures, at the six simulated observation points identified in Fig. 8. Inspection of Fig. 9, in comparison with the rainfall time series in Fig. 7, shows that the FS values gradually decrease (with increasing pore pressure) over time, falling more quickly once the cumulative rainfall depth surpassed 600 mm. Based on the FS threshold value of 1.0, slope failure is estimated to occur at the beginning of March for observation points 5 and 6 and in early April for observation point 4. The estimated failure time at observation points 5 and 6 is slightly lagged behind the most rapid movement observed at the site (i.e., early March). It is important to note that, during the entire simulation period, the FS values for observation points 1, 2 and 3 never fall below the threshold value. Clearly, the transient distributed nature of the simulated hydrologic response (see Fig. 8) is the driving force in the slope stability estimates reported here. From the beginning of the rainfall record in 1953 until the slope failure in 1973, the 1970 rainfall of 594 mm (the year the destroyed home was finished and showed immediate signs of distress) is second only to the 619 mm recorded in 1973. The demolished home at Lerida Court appears to have been doomed from the start, with failure simply a waiting game until an unusually wet winter rainy season.

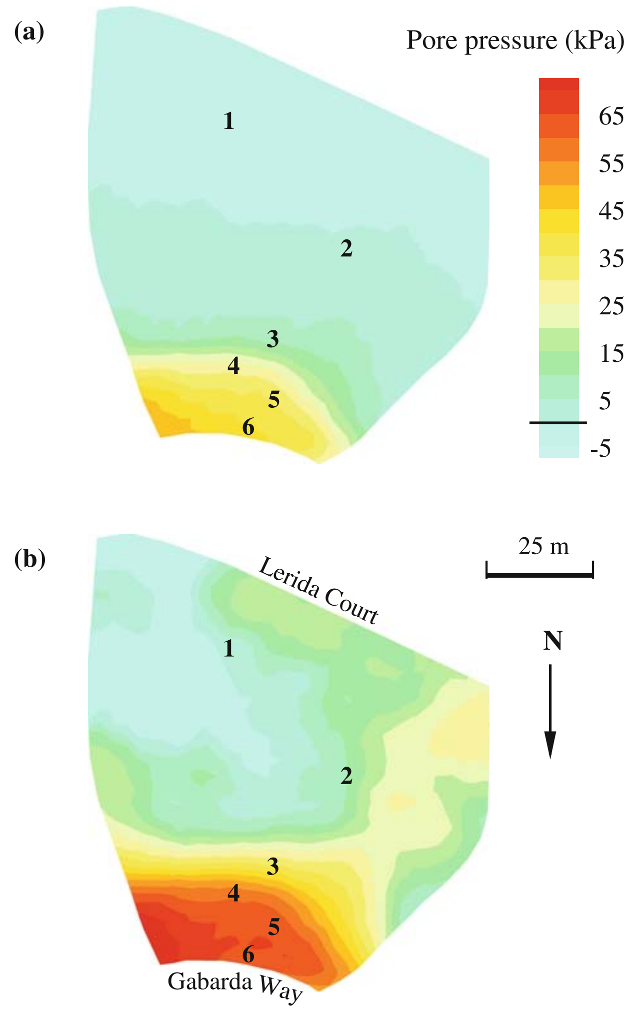


Fig. 8 Snapshots of simulated pore pressures at the soil/bedrock interface for the Lerida Court site [locations of the six simulated observation points are shown; unsaturated (negative) vs. saturated (positive) pore pressure threshold delineated on key]. **a** Initial conditions at 1 October 1972, **b** 6 months elapsed time at 1 April 1973

Fig. 9 Factor of safety estimates at the six simulated observation points shown in Fig. 8

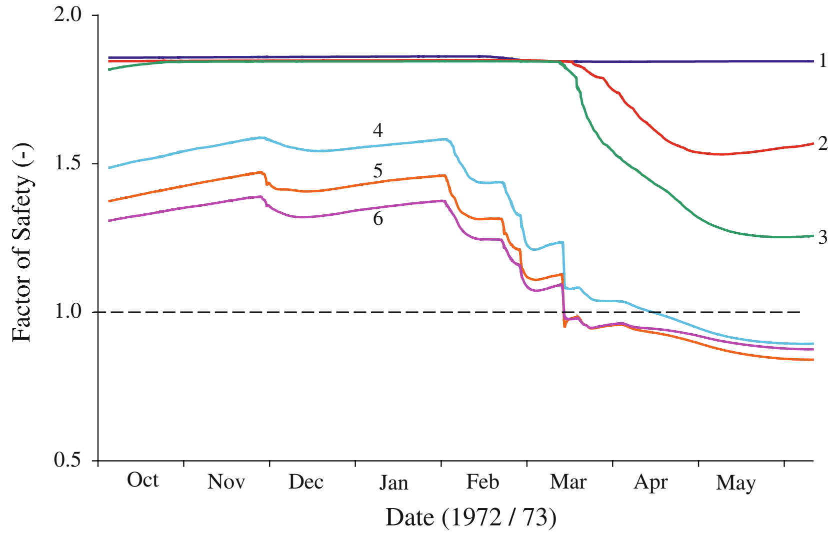


Fig. 10 Lower retaining wall (at Gabarda Way) installed in the aftermath of the 1973 Lerida Court landslide. The wall, constructed of steel I beams anchored in bedrock and connected by railroad ties wedged into place, is approximately 5-m high and 44-m long. **a** The new wall in 1974, **b** the vine covered wall in 2007

Discussion

Hydrologically driven slope failures can be complex, depending on factors, such as lithology (e.g., Evans 1982; Roering et al. 2005), heterogeneity in saturated hydraulic conductivity (e.g., Wilson and Dietrich 1987; Johnson and Sitar 1990; Vieira and Fernandes 2004), fracture flow (e.g., Montgomery et al. 2002; Ebel et al. 2008), spatial variability in geotechnical parameters (e.g., Burton et al. 1998), and unsaturated zone hydrologic response (e.g., Anderson and Howes 1985; Fourie et al. 1999; Wilkinson et al. 2002; Simoni et al. 2008; Godt et al. 2009). It should be noted that the Lerida Court failure occurred at a hydrologic and geotechnical material property contrast (i.e., the soil/bedrock interface) and that bedrock fracture flow was not observed to be a significant contributing factor to failure initiation. The small, site-scale focus of the effort reported here coupled with post-failure observations allows certain simplifying assumptions. For example, because the primary focus of this effort is the impact of variably saturated subsurface flow relative to slide initiation (with the failure plane given), the simple infinite slope model can be employed in lieu of multidimensional slope stability analysis (e.g., Bromhead et al. 2002; Borja et al. 2006) and fully coupled feedbacks between flow and deformation can be omitted from the analysis. In the spirit of the growing field of hydrogeomorphology (see Sidle and Onda 2004; Loague et al. 2006), the emphasis of this work is on demonstrating the utility of a physics-based model (InHM) for examining the dynamics of variably saturated near-surface hydrologic response as it relates to the slope stability problem.

More than 35 years have passed since the Lerida Court landslide. Figure 10 shows the retaining wall at Gabarda Way as it appeared both in 1974 and 2007 (see site B in Fig. 2 for location). The wall has helped to successfully stabilize the Lerida Court site for more than a generation. However, even with the wall in place, any consideration of a new development at or near the site where the Lerida

Court home was removed in 1973 should trigger a rigorous assessment of near-surface hydrologic response. To effectively simulate hydrologic response for the Lerida Court site as it exists today with (for example) InHM would require the development of a finite-element mesh, generated from the current (post landslide) topography that includes consideration for the retaining walls. To improve upon the deterministically crisp signature of hydrologic response and simple slope stability estimates reported here would require the acquisition of substantial hydraulic and geotechnical information from across the site.

Summary

A simulation-based characterization of the 1973 Lerida Court landslide in Portola Valley, CA, USA was the focus of this study. The hydrologic-response simulations and slope stability estimates were conducted with the comprehensive physics-based InHM and the infinite slope/factor of safety approach, respectively. Based on the available information with which to develop and parameterize the Lerida Court boundary-value problem, it was possible to successfully simulate, from a forensic perspective, a defendable hydrologic response (i.e., internally valid pore pressure distribution) that, in turn, resulted in estimates of slope failure that compare favorably with anecdotal field observations. The results herein suggest that conservative (risk averse) development in steep urban settings should include consideration for transient variably saturated hydrologic response as standard protocol.

References

Alexander D (1989) Urban landslides. Prog Phys Geog 13:157–189

Anderson MG, Howes S (1985) Development and application of a combined soil water-slope stability model. Q J Eng Geol 18:225–236

BeVille SH (2007) Physics-based simulation of near-surface hydrologic response for the Lerida Court landslide in Portola Valley, CA. MS Thesis, Stanford University, Stanford

Borja RI, Oettl G, Ebel B, Loague K (2006) Hydrologically driven slope failure initiation in variably saturated porous media. In: Wu W, Yu HS (eds) Modern trends in geomechanics. Springer, Berlin, pp 303–311

Bromhead EN, Ibsen M-L, Papanastassiou X, Zemichael AA (2002) Three-dimensional stability analysis of a coastal landslide at Hanover Point, Isle of Wight. Q J Eng Geol Hydrogeol 35:79–88

Burton A, Arkell TJ, Bathurst JC (1998) Field variability of landslide model parameters. Environ Geol 35:100–114

Carsel R, Parrish R (1988) Developing joint probability distributions of soil water retention characteristics. Water Resour Res 24:755–769

Duncan JM (1996) State of the art: limit equilibrium and finite-element analysis of slopes. J Geotech Eng 122:577–596

Dutton A, Loague K, Wemple BC (2005) Simulated effect of a forest road on near-surface hydrologic response and slope stability. Earth Surf Proc Land 30:325–338

Ebel BA, Loague K, VanderKwaak JE, Dietrich WE, Montgomery DR, Torres R, Anderson SA (2007a) Near-surface hydrologic response for a steep, unchanneled catchment near Coos Bay, Oregon: 2. Comprehensive physics-based simulations. Am J Sci 307:709–748

Ebel BA, Loague K, Dietrich WE, Montgomery DR, Torres R, Anderson SP, Giambelluca TW (2007b) Near-surface hydrologic response for a steep, unchanneled catchment near Coos Bay, Oregon: 1. Sprinkling experiments. Am J Sci 307:678–708

Ebel BA, Loague K, Montgomery DR, Dietrich WE (2008) Physics-based continuous simulation of long-term near-surface hydrologic response for the Coos Bay experimental catchment. Water Resour Res 44:W07417. doi:10.1029/2007WR006442

Ebel BA, Mirus BB, Heppner CS, VanderKwaak JE, Loague K (2009) First-order exchange coefficient coupling for simulating surface water–groundwater interactions: parameter sensitivity and consistency with a physics-based approach. Hydrol Process 23:1949–1959. doi:10.1002/hyp.7279

Evans SG (1982) Landslides and surficial deposits in urban areas of British Columbia: a review. Can Geotech J 19:269–288

FBLA (1973) Geotechnical investigation, phase III, Lerida Court and La Mesa Drive, Ladera Area, County of San Mateo (unpublished report). F Beach Leighton and Associates, La Habra

Fourie AB, Rowe D, Blight GE (1999) The effect of infiltration on the stability of the slopes of a dry ash dump. Geotechnique 49:1–13

Freeze RA, Cherry JA (1979) Groundwater. Prentice-Hall, Englewood Cliffs

Godt JW, Baum RL, Lu N (2009) Landsliding in partially saturated materials. Geophys Res Lett 36:L02403. doi:10.1029/2008GL035996

Heppner CS, Loague K (2008) A dam problem: simulated upstream impacts for a Searsville-like watershed. Ecohydrology 1:408–424

Heppner CS, Loague K, VanderKwaak JE (2007) Long-term InHM simulations of hydrologic response and sediment transport for the R-5 catchment. Earth Surf Proc Land 32:1273–1292

Hilley GE, Bürgmann R, Ferretti A, Novali F, Rocca F (2004) Dynamics of slow-moving landslides from permanent scatterer analysis. Science 304:1952–1955

Hoexter DF (1975) Engineering geology and relative stability of part of Ladera, San Mateo County, CA. MS Thesis, Stanford University, Stanford

Holtz RD, Kovacs WD (1981) An introduction to geotechnical engineering. Prentice-Hall, Englewood Cliffs

Holzhausen GR (1974) The Lerida Court landslide of 1973: renewed failure within a prehistoric landslide system in Ladera, CA. BA Thesis, Stanford University. Stanford

Johnson KA, Sitar N (1990) Hydrologic conditions leading to debris flow initiation. Can Geotech J 27:789–801

Keefer DK, Wilson RC, Mark RK, Brabb EE, Brown WM, Ellen SD, Harp EL, Wieczorek GF, Alger CS, Zatkin RS (1987) Real-time landslide warning during heavy rainfall. Science 238:921–925

Loague K, Heppner CS, Mirus BB, Ebel BA, Ran Q, Carr AE, BeVille SH, VanderKwaak JE (2006) Physics-based hydrologic-response simulation: foundation for hydroecology and hydrogeomorphology. Hydrol Process 20:1231–1237

Mirus BB, Ebel BA, Loague K, Wemple BC (2007) Simulated effect of a forest road on near-surface hydrologic response: redux. Earth Surf Proc Land 32:126–142

Mirus BB, Loague K, VanderKwaak JE, Kampf SK, Burges SJ (2009) A hypothetical reality of Tarrawarra-like hydrologic response. Hydrol Process 23:1093–1103

Montgomery DR, Greenberg HM, Laprade WT, Nasham WD (2001) Sliding in Seattle: test of a model of shallow landsliding potential in an urban environment. In: Wigmosta MS, Burges SJ (eds) Land use and watersheds: human influence on hydrology and geomorphology in urban and forest areas. American Geophysical Union, Washington, DC, pp 59–73

Montgomery DR, Dietrich WE, Heffner JT (2002) Piezometric response in shallow bedrock at CB1: implication for runoff generation and landsliding. Water Resour Res 38:1274–1292

Petro L, Vlcko J, Ondrasik R, Polascinova E (2004) Recent tectonics and slope failures in the Western Carpathians. Eng Geol 74:103–112

Roering JJ, Kirchner JW, Dietrich WE (2005) Characterizing structural and lithologic controls on deep-seated landsliding: implications for topographic relief and landscape evolution in the Oregon Coast Range, USA. Geol Soc Am Bull 117:654–668

Sas R, Sklar LS, Eaton LS, Davis J (2008) A method for developing regional road-fill failure hazard assessments using GIS and virtual fieldwork. Environ Eng Geosci 14:221–229

Selby MJ (1993) Hillslope materials and processes. Oxford University Press, Oxford

Shlemon RJ, Wright RH, Montgomery DR (1987) Anatomy of a debris flow, Pacifica, California. In: Debris flows/avalanches: process, recognition, and mitigation, vol VII. Reviews in Engineering Geology, Geological Society of America, pp 181–199

Sidle RC, Onda Y (2004) Hydrogeomorphology: overview of an emerging science. Hydrol Process 18:597–602

Simoni S, Zanotti F, Bertoldi G, Rigon R (2008) Modelling the probability of occurrence of shallow landslides and channelized debris flows using GEOtop-FS. Hydrol Process 22:532–545

Smith TC, Hart EW (1982) Landslides and related storm damage, January 1982, San Francisco Bay region. Calif Geol 35:139–152

Spiker EC, P Gori (2003) National landslide hazards mitigation strategy: a framework for loss reduction. US Geological Survey Circular, p 1244

Stark TD, Eid HT (1998) Performance of three-dimensional slope stability methods in Practice. J Geotech Geoenv Eng 124:1049–1060

Taylor FA, Nilsen TH, Dean RM (1975) Distribution and cost of landslides that have damaged manmade structures during the rainy season of 1972–1973 in the San Francisco Bay region, California. US Geological Survey Miscellaneous Field Studies Map MF-679, scales 1:500,000, 1:1,000,000

van Genuchten MTh (1980) A closed-form equation for predicting the hydraulic conductivity of unsaturated soils. Soil Sci Soc Am 44:892–898

VanderKwaak JE (1999) Numerical simulation of flow and chemical transport in integrated surface-subsurface hydrologic systems. PhD Dissertation, University of Waterloo, Waterloo

VanderKwaak JE, Loague K (2001) Hydrologic-response simulations for the R-5 catchment with a comprehensive physics-based model. Water Resour Res 37:999–1013

Vieira BC, Fernandes NF (2004) Landslides in Rio de Janeiro: the role played by variations in soil hydraulic conductivity. Hydrol Process 18:791–805

Vlcko J (2004) Extremely slow slope movements influencing the stability of Spis Castle, UNESCO site. Landslides 1:67–71

Wilkinson PL, Anderson MG, Lloyd DM (2002) An integrated hydrological model for rain-induced landslide prediction. Earth Surf Proc Land 27:1285–1297

Wilson CJ, Dietrich WE (1987) The contribution of bedrock groundwater flow to storm runoff and high pore pressure development in hollows. In: Beschta RL, Blinn T, Grant GE, Ice GG, Swanson FJ (eds) Proceedings of the international symposium on erosion and sedimentation in the Pacific Rim. IAHS Publication 165, pp 49–59

Wong F (1984) Uncertainties in FE modeling of slope stability. Comput Struct 19:777–791